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Soil-Pile Interaction of Pile embedded in Deep Layered Marine Sediment under Seismic Excitation

Abstract

This paper investigates the soil-pile interaction of a pile embedded in a deep multi-layered soil under seismic excitation considering both kinematic and inertial interaction effects. A comprehensive three dimensional finite element model is developed and validated using existing results in the literature. The response of the pile in the deep multi-layered soil profile is investigated with respect to pile head response, deflection modes and maximum deflections along the pile. Results show that the pile exhibits complex deflection patterns and that the pile response is influenced by the properties of both the soil profile and the seismic excitation. It is also evident that kinematic interaction effects have a greater influence on the pile response than the inertial interaction effects.

Keywords: soil-pile interaction, seismic excitation, multilayered soil, kinematic and inertial

1.0 Introduction

Response of laterally load piles is a complex phenomenon of soil-pile interaction as the lateral response of the pile depends on the resistance provided by the surrounding soil and soil resistance in turn depends on the pile deflection. Complexity of this phenomenon depends on many factors such as loading type and soil profile. Researches have been carried out to investigate soil-pile interaction problems using different methods of analysis, while the beam-on-foundation method being the popular method due to its simplicity. However, this method is essentially one dimensional and hence unreliable in representing the actual soil-pile interaction, which is three dimensional in nature. In addition the springs used in this simple method to represent the soil, lacks the proper representation of soil continua which influence the pile deflections under lateral loads. Beam on Dynamic Winkler Foundation (BDWF) method has proved to be efficient in predicting the pile response under dynamic conditions, but it is limited to linear or linear equivalent pile and soil behaviour. On the other hand, finite element (FE) techniques provide a promising means of modelling and analysing the soil-pile system in three dimensional domain accounting for complex nonlinearities of the soil pile system.

Irrespective of the method of analysis, research on soil pile interaction has been carried out considering kinematic interaction effects on the pile response [1, 2, 4, 5, 6], kinematic and inertial interaction effects on the pile response [3,7,8], slipping and gapping in soil pile interface [1,2], soil nonlinearity [8,9] and liquefaction [3]. Also most of the studies on soil-pile interaction had been carried out in the frequency domain [2, 4, 6, 7, 10] and only few are in the time domain [1, 11].

However, most of the studies carried out on the soil-pile interaction analysis are based on homogeneous soil profiles consisting of relatively stiff soils and the piles considered are of limited depths, typically around 10m. Therefore, the studies carried out so far are lacking the practical significance of soil-pile interaction.

Irrespective of the research carried out in the area of soil-pile interaction, design of piles for seismic excitations still remains challenging in actual engineering practise. In this case, a pseudo-static procedure is carried out where, seismic forces and moments are applied at the head of the pile as static loads and analysis is carried out to check the allowable design values. This essentially neglects the additional loads that act on pile due to the lateral deflections caused by the movement of the surrounding soil under seismic wave propagation. Even though some researchers have suggested simplified pseudo-static approaches [12,13] to account for kinematic effects, such methods are not popular among engineering practitioners.

Though some design codes [14] suggest that kinematic interaction effects should be considered in the design of pile foundations, there are no deterministic methods or validated techniques that can be used in the pile design process to capture the responses caused by the kinematic interaction effects. This research addresses this issue and proposes a comprehensive three dimensional finite element procedure with validated techniques that can capture the pile response under the combined effects of kinematic and inertial actions in time domain. The developed model is then used to investigate the soil-pile interaction behaviour of deep pile foundation embedded in a multilayered soil profile which has a soft soil layer. The influence of the soil types in the different soil layers is captured in this paper through the different vibration modes under seismic response. This feature, to the best of knowledge of the authors, has not been captured before.

2.0 Development of Numerical Model

In this study the soil-pile system is modelled in the three dimensional domain using the general purpose finite element software, "ABAQUS" [15]. The governing equation of the system is given by

$$[M]\{\ddot{u}\} + [C]\{\dot{U}\} + [K]\{U\} = \{F(t)\} [1]$$

where, [M], [C] and [K] are the mass, damping and stiffness matrices respectively and $\{\ddot{U}\}$,$\{\dot{U}\}$and $\{U\}$are the acceleration, velocity and displacement vectors while $F(t)$ is the forcing function.

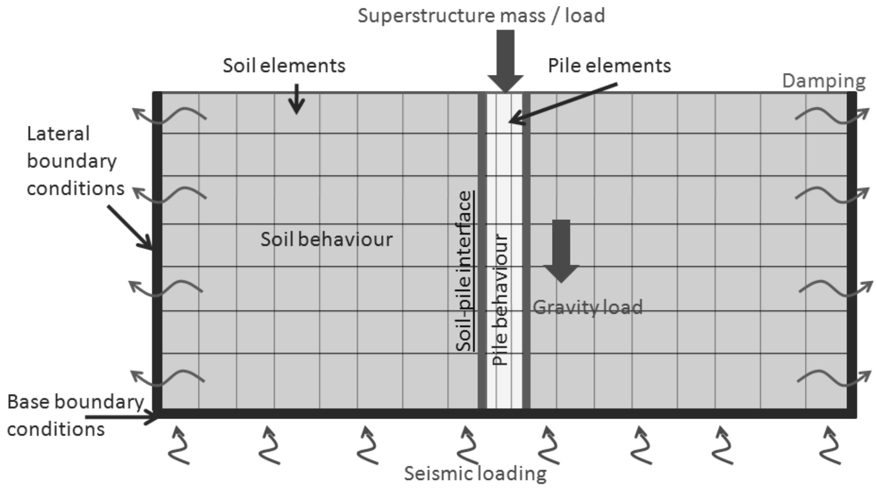


Figure 1: Model Development Basic Components of the FE Model

The basic components of the developed Finite Element model, are shown in figure 1 a) schematically, and are described in the subsequent sections.

2.1 Elements and Mesh Sizes

Contrary to past research, where eight node brick elements were used to model both soil and pile, this study instead uses eight node tri-linear displacement and pore pressure element type (C3D8RP) for the soil. The suitability of this element type to model soil in contrast to traditional brick element type has been verified by real applications [16] and used in soil analyses successfully [15]. This type of element can overcome the excessive settlements of soils under gravity which is a problem associated with traditional eight node brick elements, especially when soft soils are involved. Also the lateral pressure of soil can be modelled precisely by introducing the "lateral earth pressure coefficient, k_0". Eight node linear brick elements (C3D8) are however used to model the pile as in past research.

The pile was considered as a cantilever beam fixed at the base as the piles considered in the present study is considered as fixed at base. A horizontal load was applied at the top and deflections were obtained at different heights using FE method and then compared with the theoretical values for different mesh sizes. The mesh size for which the deflections closely matched the corresponding theoretical value was selected for use in further analysis.

The subdivisions in the vertical direction of the soil were kept constant within a soil layer to distribute the waves evenly in the soil profile. The maximum element size for soil was maintained at a value less than one-fifth to one-eighth the shortest wave length (λ) to acquire the required accuracy [17]. Here, $\lambda = V_s / f$, in which V_s is the shear wave velocity and f is excitation frequency. The maximum frequency considered here is 20 Hz.

2.2 Material models

Selection of proper constitutive models for the material behaviour is essential in numerical modelling. Similar to past research, this study also assumes that the pile behaviour is linear elastic throughout the analysis [5,6]. Most of the past research on soil-pile interaction used elastic material models to simulate the soil behaviour. But soil in most instances shows nonlinear behaviour and hence plasticity should be incorporated. A simple elastic-perfectly plastic model can simulate the behaviour of soil with a sufficient accuracy though there are different ways to incorporate the plastic behaviour of soil. These types of material models have been successfully used in the literature in wave propagation problems [5]. The Mohr-Coulomb model which suggests that the yielding begins when the shear stress τ and normal stress σ_n satisfy the following equation was used in the present study.

$$\tau = C + \sigma_n . tan\emptyset \quad [2]$$

In the above equation, C is the cohesion and $\emptyset$ is the friction angle of the soil. The yield criterion of the Mohr-Coulomb model is defined as:

$$f = (\sigma_1 - \sigma_3) - (\sigma_1 + \sigma_3).\sin\phi - 2C.\cos\phi = 0 \quad [3]$$

where, σ_1 and σ_3 are maximum and minimum principal stresses.

2.3 Soil-pile interface

Here, the surface-based interaction technique available in ABAQUS was used to model the soil-pile interface. This involves interaction between two surfaces which are defined based on their rigidities. The more deformable surface is defined as slave surface while the one with the greater rigidity is defined as the master surface. Master and slave surfaces for this study are surfaces of the pile and the soil respectively. The interaction behaviour of these two surfaces was defined in terms of normal behaviour and tangential behaviour. Normal behaviour was modelled as "hard" contact behaviour. This approach allows any pressure to be transmitted between surfaces if they are in contact. The surfaces separate if the contact pressure reduces to zero and hence will not transmit any tensile stresses. Tangential behaviour was based on the Mohr-Coulomb friction model and accordingly two contacting surfaces can carry shear stresses up to a certain level before they start sliding relative to one another. The Coulomb friction model defines this critical shear stress τ_{crit}, at which the sliding of surfaces starts as a fraction of the contact pressure, P between the surfaces ($\tau_{crit} = \mu P$), where μ = coefficient of friction.

2.4 Loading steps

Unlike in the analysis of most structures, where the analysis begins with a stress free mesh, in buried structures the response depends on the history of loading at the in-situ condition. Therefore it is important to simulate the in-situ conditions before applying any seismic loads to the model. In the present work, loading to the model was applied in two consecutive steps; geostatic and dynamic loading steps.

In the geostatic loading step geostatic stress condition was simulated by applying gravity load to the system together with a predefined stress field which is applied to the soil mesh. For this, vertical stresses (calculated at a point by summing up the products of the unit weight and height of each soil layer above that point) were specified at two points and the change of vertical stress between those two points was assumed linear. To define the horizontal stress in the soil, the lateral earth pressure coefficient was defined so that the program could calculate the horizontal stress in soil. The program creates a force equilibrium system where the in-situ stresses are calculated. These in-situ stresses are in equilibrium with external forces under the prescribed boundary condition and produce negligible deformations.

In the present study, dynamic loading was applied at the bedrock level as a horizontal acceleration and responses were measured in the direction of ground shaking.

2.5 Boundary conditions

In dynamic analysis of soil-pile interaction the surrounding soil strata is considered as infinite in the horizontal direction. It is therefore important to avoid wave reflection at the vertical boundaries. To do this some researchers suggest that energy transmitting boundaries can be used [18]. This included attaching dashpots or Kelvin elements (dashpot and a spring attached parallel to each other) at the lateral boundaries and many researchers tried to find coefficients for these dashpot and spring constants. However, in order to use those coefficients, the systems considered have to satisfy certain conditions such as having a homogeneous soil medium [18], linear elastic soil medium [18], and information on type of loading and types of waves considered in the analysis [18,19]. Whenever these requirements are not met, such transmitting boundaries cannot be used. In such situations, when the system is subjected to a seismic loading, some authors have suggested [20,21] that either “free horizontal motion and zero vertical motion” boundaries or repeating boundaries [21] are ideal to use as they do not require any conditions to satisfy. The present study has adopted the first type of boundary condition for which the static active failure caused by the movement of soil in the horizontal direction at the vertical lateral boundary has to be prevented by applying lateral earthpressure [21]. However, when using this boundary condition, lateral boundaries should be located at a distance far enough from the area of interest so that wave decay caused by energy dissipation (in the material) during propagation and reflected waves will not affect the calculated response in the area of interest [21]. In this case, a trial and error process was carried out to find out the location for the lateral boundaries, so that pile response is not affected furthermore with the change of the position.This type of a simple boundary condition is adopted in many commercial geotechnical software in dynamic analyses [22]. This lateral boundary condition was applied in the geostatic loading step and extended to the dynamic loading step.

Boundary condition at the base of the model depends on the loading condition. During the geostatic loading step, the base is considered fixed. But in the dynamic loading step, it was free to move in the horizontal direction. During this step, the selected seismic excitation was applied to the base in the horizontal direction.

2.6 Damping

In soil-pile interaction problems damping occurs in both the pile foundation and the soil. However, damping in pile is considered negligible when compared to that of soil. Due to this reason most of the studies conducted to investigate soil-pile interaction problems, didn't consider the damping in pile foundation, but only the damping in soil [5,6]. This study also assumed that the damping occurs only in the soil, neglecting the damping in pile foundation. Material damping in soils is considered to be achieved mainly through viscous damping. Therefore, traditionally, when computing material damping in soils, mass proportional damping is neglected and damping of the soil is achieved through stiffness proportional material damping. Damping matrix is hence reduced to a single matrix, which is proportional to the stiffness matrix.

Therefore, damping of the soil-pile system is achieved through stiffness proportional material damping and it is assumed to be constant throughout the analysis. This type of damping has been successfully used in dynamic soil-pile interaction analysis in the literature [5,6]. The damping matrix is given by equation 4.

$$[C] = \beta[K] \quad [4]$$

where, [C]= damping matrix, [K]=stiffness matrix and β=damping coefficient.

In this case, $\beta = 2\xi/\omega_0$, where, ω_0 is the predominant frequency of loading and ξ is the material damping ratio which is assumed to be 5%. Predominant frequency is obtained from a Fourier spectrum for the input wave.

2.7 Representation of the Superstructure

Representing the superstructure becomes challenging when real structures are considered, especially when the superstructure is massive. In some analysis carried out in soil-pile interaction problems, the whole structure is modelled on top of the pile (coupled system) [7, 23]. However, when a multi-storey building is considered the modelling techniques can be challenging and can increase the computational time and cost drastically. In such situations, the common practice is to model the superstructure using lump-mass model. Even though this is the common tendency, Liyanapathirana and Poulos [3] suggested that attaching the superstructure mass at the cap level of the pile foundation provides sufficient accuracy, at least for initial pile design. Since the main focus of this study is the soil-pile interaction under seismic excitation of piles that support a multi-storey building, the method suggested Liyanapathirana and Poulos [3] is used.

3.0 **Validation of the Numerical Model**

To ensure the proper behaviour of the developed numerical model with the selected modelling techniques under lateral loading conditions, validation was carried out for both static and dynamic loading conditions. Results from previous studies reported in the literature were used for this purpose.

A 10m long socketed pile with a square cross-section of 0.5m × 0.5m is used as in a previous study [5]. The pile is considered as a linear elastic element with a density of 2300 kg/m3, Young's modulus of 20 GPa and Poisson's ratio of 0.25. Soil used in this validation had a density of 1203 kg/m^3, Young's modulus of 20MPa, Poisson's ratio of 0.45, cohesion of 34kPa and an internal friction angle of 16.5^0. This data was obtained from the soil-pile interaction study carried out by Bentley and Naggar [5] and used in both types of validation carried out in this study to maintain consistency.

3.1 Validation under Static Loading

As the first step of the validation of the developed numerical model, static response analysis was carried under a lateral load applied at the head of the pile. Unlike under axial loading, the response of the pile under lateral loads is affected by the soil-pile interactions and depends greatly on the lateral confinement provided by the surrounding soil. Therefore, the validation under static loading conditions ensures that proper confinement is provided by the surrounding soil. Validation under static loading condition was carried out by loading the pile in an identical manner as in the study by Bentley and Naggar [5] and under similar conditions, i.e. with and without considering the gap formation between pile and soil. The study conducted by Bentley and Naggr [5] was also based on a three dimensional finite element model. However, the developed numerical models in the present study and in the study by Bentley and Naggar [5], have differences in type of elements used to model soil and the type of lateral boundary conditions used. A horizontal load was applied at the top of the pile and the pile response at the top was obtained. When the separation was not allowed between soil and pile, there was a higher resistance to deformation. However, when the separation was allowed, comparatively higher deflections were observed specially under higher loads. Results obtained here were then compared with those obtained by Bentley and Naggar [5]. Figure 2 shows the results for both conditions and it is evident that the present results compare well with those from reference [5]. The small difference observed between the two sets of results are probably caused by the differences in the modelling techniques used.

a) Without gap formation b) With gap formation

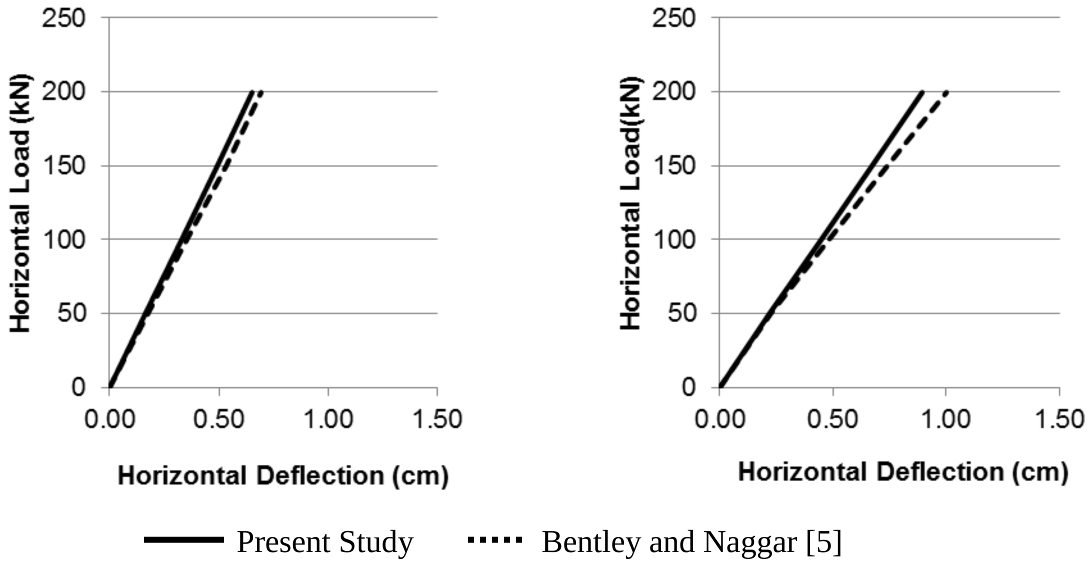
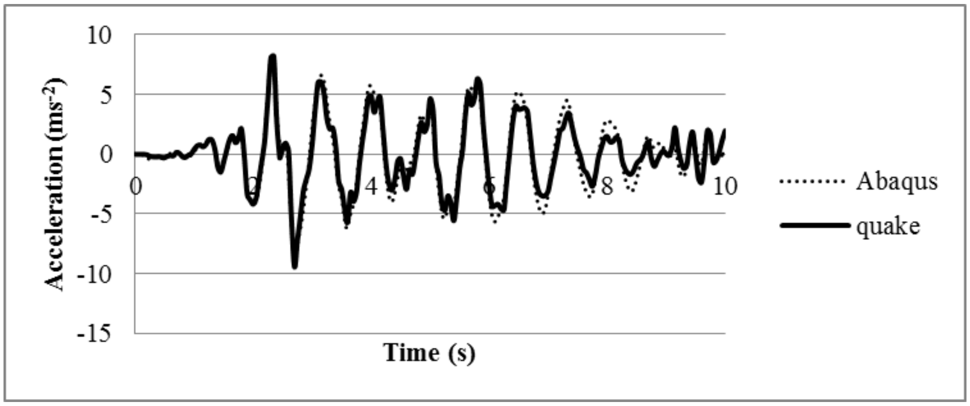


Figure 2: Response of single pile under static loading

3.2 Validation under Dynamic Loading

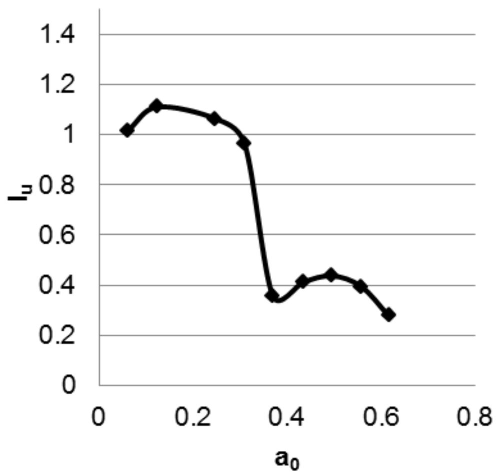
Validation was also carried out to ensure the proper behaviour of the developed model under dynamic loading conditions. In dynamic analysis, some additional model parameters are activated which are not considered during static loading conditions such as damping and the seismic excitation given at the base of the model. Dynamic validation was carried out in two steps: a) for free field motion and b) for soil-pile system with a base excitation considering (i) kinematic interaction effects only and (ii) combined kinematic and inertial interaction effects

The free field motion validation ensures the proper wave propagation in the soil medium under a seismic excitation, which governs the motion of the system. The validation of the soil-pile system ensures the proper pile response at different frequencies when subjected to a sinusoidal input motion at the base of the soil-pile system.

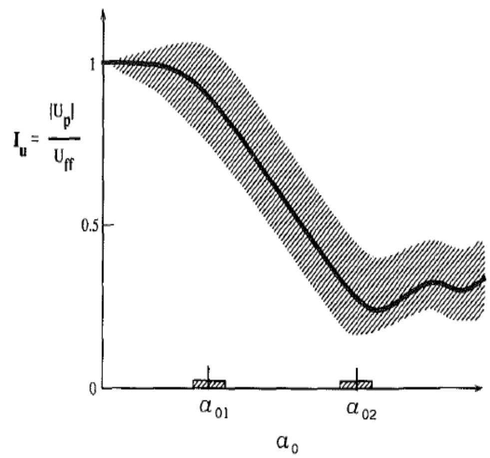


a) Free field response

b.1) Present study

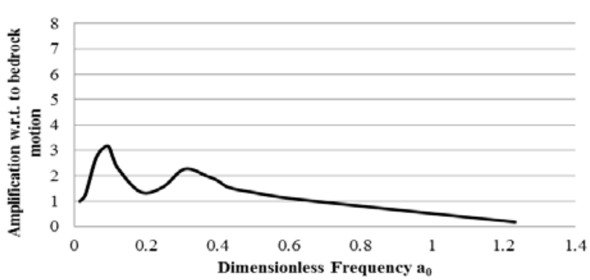


b.2) Study by Fan et al ([24])

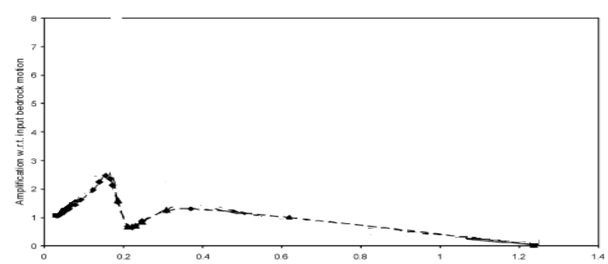


b) validation for kinematic effects

c.1) Present study



b.2) Study by Maheshwari et.al [7]



c) Validation for kinematic and inertial combined effect

Figure 3 : Validation for Dynamic Loading a) Free field response b) validation for kinematic effects c) Validation for kinematic and inertial combined effect

Validation of free field motion was carried out only for the soil profile in the absence of the pile. Here, an excitation was given as a displacement-time history to the base of the profile in order to simulate seismic waves. The displacement-time history used here was from the seismic data of the El-Centro earthquake*. Free field for the same soil profile and base excitation were then obtained using "Geostudio-QUAKE" software which can be used directly for such analysis. Free field responses obtained from both analyses are compared in Figure 3a which shows a very good agreement, between the two sets of results with the same maximum acceleration and minor differences in the other parts of the response. These discrepancies are possibly caused by the differences in modelling techniques used, as the "Geostudio-QUAKE" is a commercial software used purposely for this types of simulations, where as the present study uses the numerical model developed with the general purpose finite element software ABAQUS. One of the main difference that can be noted is the method used to simulate the damping in the system. In the "Geostudio-QUAKE", damping was given as a percentage with respect to viscous damping of soil, where as in ABAQUS material damping is defined by specifying a damping coefficient as described in section 2.6.

Validation for the kinematic interaction effects was carried out in accordance with the study done by Fan et. al. [24] by giving the base of the soil-pile system a sinusoidal excitation which is described in equation [5]. Fan et.al. [24] carried out this study on the kinematic soil-pile interaction problem in the frequency domain, based on the formulations proposed by Kaynia and Kausel [25] using the boundary-integral-type formulation.

$$x(t)=A\sin(\omega t) \qquad [5]$$

where, x(t) is the displacement, A is its amplitude, ω is the angular frequency and t is the time. For the comparison of results, two dimensionless parameters were defined, namely; Kinematic Displacement Factor I_u and Dimensionless Frequency a_0.

$$I_U = |U_p|/U_{ff} \qquad [6]$$

Where U_p =pile response at top U_{ff}= amplitude of free field motion

$$a_0 = \frac{\omega.d}{v_s} \qquad [7]$$

Where ω=circular frequency of loading, d=pile diameter (width in this case), V_s=shear wave velocity of soil

Figure 3a.1 shows the results of the present study up to a frequency of 10Hz (a_0≈0.6) and figure 3b.1 shows the idealized general shape of kinematic displacement factor vs. dimensionless frequency proposed by Fan et al.[24].

Both studies show that there is a low frequency region ($0<a_0<a_{01}$) in which I_u≈1. This means that the pile follows the deformation of the ground for this frequency range. Fan et al. [24] observed that a_{01} has a value of 0.2~0.3, which is true for the present study as well. Then there is an intermediate region ($a_{01}<a_0<a_{02}$), where I_u declines rapidly with the frequency. Finally there is a relatively high frequency region ($a_0>a_{02}$) in which I_u fluctuates around a mean value of about 0.4. Fan et al. [24] observed that a_{02} can be 5 to 10 time the natural frequency of the soil deposit for a homogeneous soil profile. In the present study a_{02} is observed when it is five time the natural frequency of the soil layer. From the similarities in the results of the two studies, it is evident that reasonably good validation has been achieved for this step.

For the validation under the combined interaction effects, a mass of 5400kg was attached at the head of the pile, and base of the soil pile system was given a sinusoidal shake as described in equation 5.Maheshwari et al [7] also used a similar type of system with similar properties in the model.

Figure 3c shows the results obtained from the present study for the pile head amplification with respect to the bedrock motion at different frequencies. The dimensionless frequency (a_0) in this graph is also similar to the a_0 described in equation 7. Results from the present study were then compared with those from the study carried out by Maheshwari et al [7]. The patterns of the curves in both studies are similar even though the values obtained from the present study are about 10% higher than the values obtained by Maheshwari et al. Eventhough both studies used 3-D FE techniques, there are several differences in the modelling techniques used. The main reason for the slight differences in the results is however believed to be due to the different techniques used to incorporate inertial effects.Maheshwari et al [7] modelled the whole superstructure without applying any simplifications, where as the present study used a structural mass attached at the head of the pile. Overall, results from the present study agree reasonably well with those from the study by Maheshwari et al. [7].

These validations provide confidence in the modelling techniques used in the present study which will then be applied to treat the seismic response of a deep pile embedded in a layered soil.

3.3 Computational Time

Bentley and Naggar [5] stated that static solution processing time averaged between 5 and 45 minutes. In their study dynamic solutions were extremely time demanding for a similar model. In their study, for a 20 s earthquake processing took approximately 10 days on a personal computer. However, with the use of supercomputers, the processing time is significantly reduced in the present study. Static analysis processing time is 30s on average and dynamic analysis processing time for a 20s earthquake is 8 hours on average.

4.0 Application: Seismic Response of Pile in Deep Multi-layer Soil Profile

4.1 Problem Definition

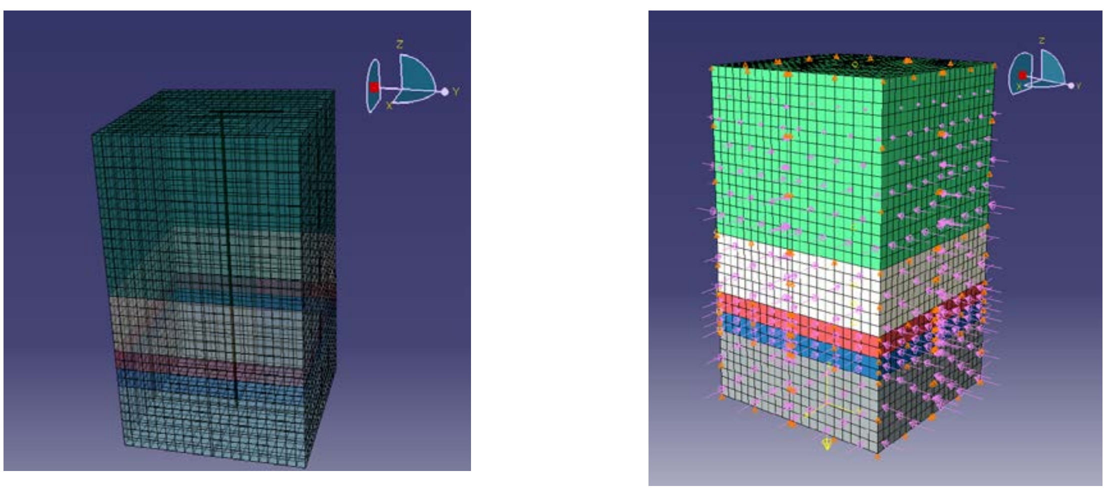
As there are several limitations in carrying out experimental studies to predict the soil-pile interaction behaviour under lateral dynamic loading conditions, numerical simulations have become a popular method to simulate the pile behaviour under such situations. However, only a few studies have been carried out to investigate the pile soil interaction under seismic excitations in the time domain, and they are limited to homogeneous soil profiles.

Among the numerical methods available to solve soil-pile interaction problems, the Finite Element Method is considered to be a very viable method compared to the popular Winkler or the beam on foundation method, which lacks the realistic modelling of the pile foundation together with the surrounding soil in three dimensional domain.

This paper describes the study carried out to investigate the seismic response of a pile embedded in a real (existing) soil profile obtained from a site investigation report for Melbourne Docklands, Australia [25]. Using the Cone Penetration Test results and Standard Penetration Test results in the report, soil layer thicknesses were obtained and the same data was used to estimate the soil layer properties using empirical formulae given in handbooks [26] and technical reports [27]. This soil profile resembles a deep multilayered soil profile with a soft soil layer at the top, as typically found in marine environments and this soil profile consists of 5 layers with increasing stiffness with depth. The soil properties are shown in table 1.

Layer No.	**Layer Thickness (m)**	**Density (kg/m^3)**	**Young's Modulus (MN/m^2)**	**Poisson's Ratio**	**Friction Angle(^{0})**	**Cohesion (kN/m^2)**
1	16	1631	10	0.4	0	39
2	6	1835	15	0.4	0	59
3	2	1886	21	0.4	0	83
4	2	1937	63	0.3	35	0
5	7	1937	248	0.3	50	0

Table 1: Soil Properties

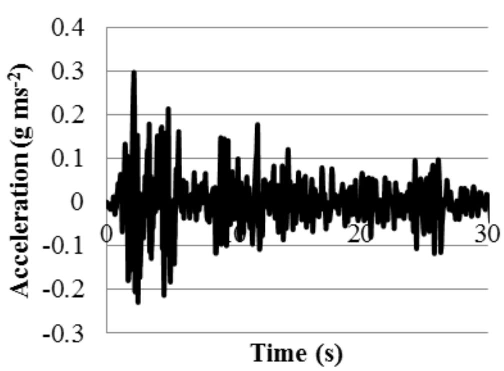


a) b)

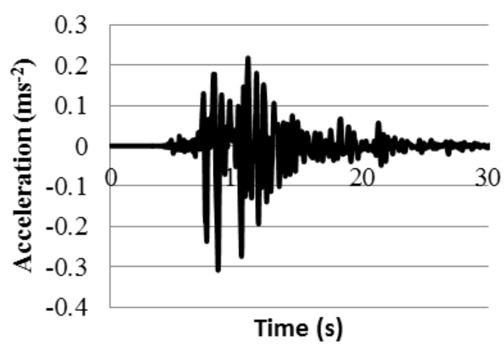
Figure 4: Screen Shots of the Developed Model a) Pile embedded in the layered soil profile b) Soil-pile system with the lateral boundary conditions

A precast concrete pile of 0.25m x 0.25m cross-section and 33m length was used in the investigation. The reason for selecting this pile size for the analysis is that it is a standard precast pile extensively used in practice to support multi storey buildings (Figure 4). Young's Modulus and Poisson's ratio of the pile were taken as 36GPa and 0.15 respectively. In this analysis, soil was considered as an elastic-plastic material, while the pile was assumed to behave linear elastically. When incorporating inertial interaction effects, a super structural mass of 100,000 Kg was attached to the head of the pile.

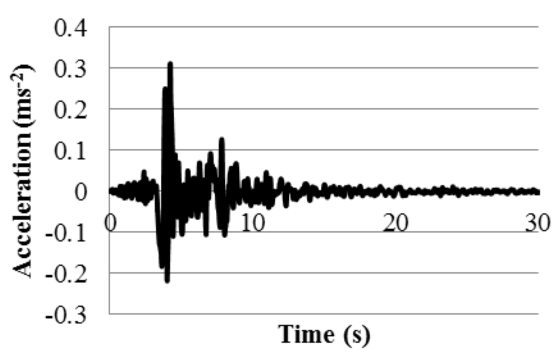
Generally earthquakes have different characteristics with respect to peak acceleration, dominant frequency content, duration of strong motion and total duration of excitation. In the present study, three earthquakes El-Centro, Kobe and Northridge were selected to be used in the analysis. The original earthquake records of El-Centro, Kobe and Northridge have peak accelerations of 0.3g, 0.8g and 0.8g respectively. However, all these seismic records were scaled to have same peak ground acceleration (0.3g) to facilitate comparison and to suit Australian seismic conditions (figure 5). Even though all the three earthquakes are scaled to the same maximum acceleration, they differ from each other with respect to the frequency content, dominant frequencies and excitation characteristics. The El-Centro earthquake has a considerable amount of shaking for a relatively longer period, where as Kobe and Northridge earthquakes show a strong excitation over a short period of time, and the Northridge earthquake also shows an abrupt reduction in acceleration.



a) El-Centro Earthquake



b) Kobe Earthquake



c)Northridge Earthquake

Figure 5: Scaled Earthquake Records; a)El-Centro b) Kobe c)Northridge

4.2 Results and discussion

The subsequent section of this paper describes the pile head response, effect of soil stiffness on pile response, pile deflection modes and maximum pile deflections to illustrate the behaviour of the deep pile foundation in marine sediments in time domain by considering kinematic effects and combined kinematic and inertial effects.

4.2.1 Pile Head Response

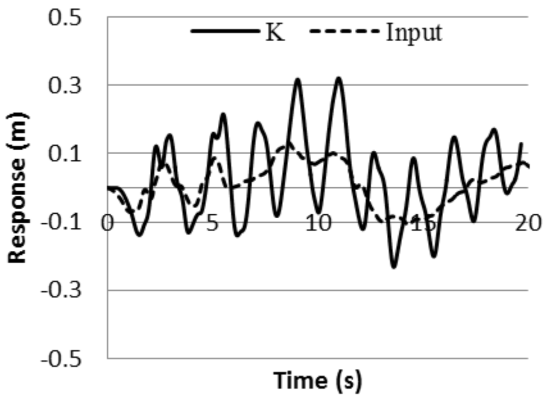
4.2.1.1 Pile Head Response Considering Kinematic Interaction Effects

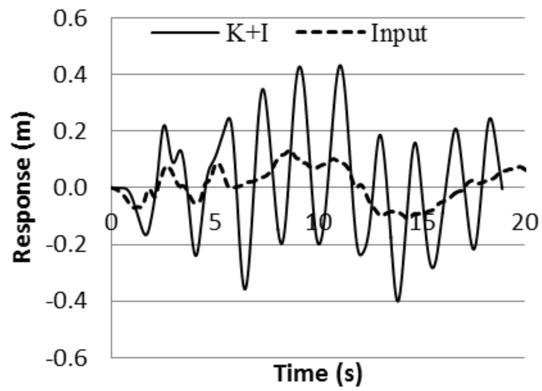
Figure 6 shows the time histories of the pile head response when subjected to different seismic excitations, along with the input excitations. Under the El-Centro earthquake, the pile head response follows the pattern of input motion where, peak response occurs near the peak input motion. In general, when the soil-pile system is subjected to the El-Centro earthquake, the pile head response shows an amplification of three times the input motion, giving a maximum response of about 0.3m.

a)El-Centro earthquake

a.1) Kinematic

a.2) Kinematic + Inertial

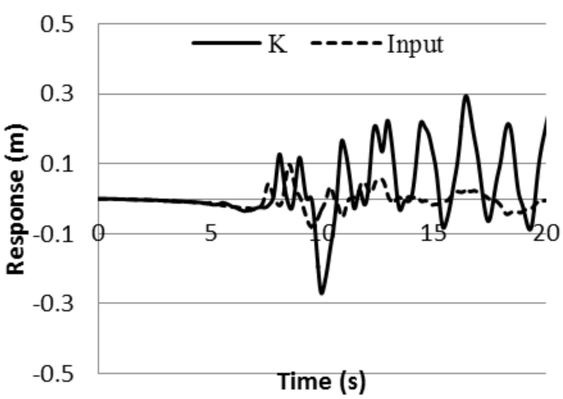


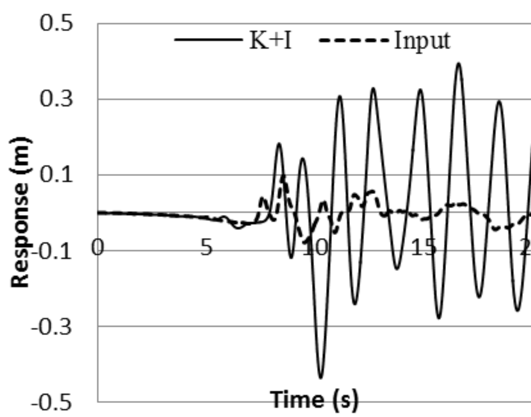


b)Kobe earthquake

b.1) Kinematic

b.2) Kinematic + Inertial





c)Northridge earthquake

c.1) Kinematic

c.2) Kinematic + Inertial

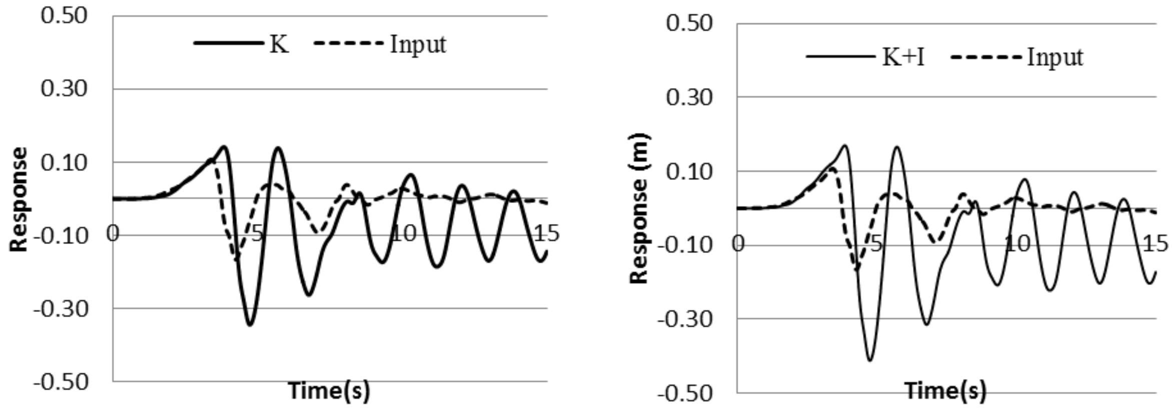


Figure 6: Comparison of pile head response under kinematic interaction effects and kinematic and inertial combined effects a)El-Centro earthquake b)Kobe earthquake c)Northridge earthquake

However, when the soil-pile system is subjected to either Kobe earthquake or the Northridge earthquake, pile head response does not follow the pattern of the input motion. It should be noted that both these earthquakes show significant excitation during a short period of time and then cease. In these two cases, pile head response follows the pattern of the input motion during the first 14s and 9s of the Kobe and Northridge earthquake records respectively, with a phase lag. Even after the excitation ceases, motion of the pile head continues to occur in both cases and these deflections oscillate about an axis which is different to the pile's original vertical axis. Such oscillations, about a displaced axis, occur due to the surrounding soil undergoing plastic deformation and its inability to return to its original position, unlike soil that has elastic behaviour.

Under the Kobe earthquake, the maximum pile head response is 0.3m which is three times the maximum input motion. However, two peak values can be observed in this case, one after the peak input motion and other one after the excitation ceases and the oscillation continues to occur about a different axis. This relatively high amplitude deflection is caused by the close compliance of the natural frequency of the soil-pile system (1.4Hz) with the dominant frequency of the input excitation (Dominant frequency of the Kobe earthquake = 1.4Hz)

Under the Northridge earthquake, the maximum pile head response occurs just after the maximum input value and is also about three times the input motion (0.3m). Even though the oscillations continue after the cessation of the excitation, they have low amplitudes compared with the behaviour under Kobe earthquake.

The behaviour described above cannot be observed when the soil-pile system is subjected to the EL-Centro earthquake and this is probably due to the pattern of loading. In contrast to Kobe and Northridge earthquakes, where shaking is sudden and then comes to a halt, the El-Centro earthquake shows a significant shaking over the entire time period considered herein.

4.2.1.2 Pile Head Response Considering Combined Kinematic and Inertial Interaction Effects

Figure 6 also shows the pile head response under the combined kinematic and inertial effects, compared with that due only to kinematic interaction effects.

As seen from this figure, the pattern of pile head motion under the combined kinematic and inertial effects is almost similar to the pile head response pattern under kinematic interaction effects for all three applied seismic excitations. Nevertheless an amplification and a small phase lag can be observed in all three cases considered here, due to the presence of structural mass attached to the pile head. Under the combined kinematic and inertial effects, the pile is subjected to a maximum response of about 0.4m whereas under the kinematic interaction only, the maximum response was about 0.3m. Hence the inclusion of the inertial effect has increased the maximum pile head response by about 33%.

4.2.2Effect of Soil Stiffness on Pile Response

Kinematic soil-pile interaction problems generally deal with the deviation of pile motion with respect to the input motion. If this deviation is negligible, it might be reasonable to carry out the analysis of piles according to pseudo-static analysis which neglects the kinematic soil-pile interaction effects. However, such scenarios are limited in real life applications and can be applied if the piles are short and embedded in relatively stiffer soils. When long and slender piles are considered, kinematic interaction effects caused by the movement of surrounding soils can greatly affect the pile deflections along its depth. The amount of deflection can be influenced by the stiffness difference between pile and soil and the stiffness of the soil layers. This section presents the time histories of the seismic response of pile at different layers of the soil profile in which the pile is embedded (Figure 7). The influence of the stiffness of the soil layers on the pile response is obtained by considering the pile response at the mid depth of each soil layer. Input motion was selected as a baseline to compare the variation in pile response due to the stiffness difference between pile and soil. For the clarity, figure 7 shows the pile response at of three layers (layer-1, layer-2 and layer5) out of five layers considered in this study, which has increasing stiffness.

In figure 7, “K -0.25” and “KI -0.25” refer to the response of the 0.25m x 0.25m pile under kinematic effects only and under combined kinematic and inertial effects respectively. As seen in this figure, the portion of pile embedded in the softest layer shows the most significant deviation from the input motion under all three seismic excitations. However, as the stiffness of the soil increases with depth, the deviation of pile response from the input motion reduces and becomes almost zero in the stiffest layer. From these results it can be concluded that considering only the inertial interaction effects, as normally done in practice, may not be adequate for carrying out seismic analysis of pile foundations when they are embedded in softer soils**.**

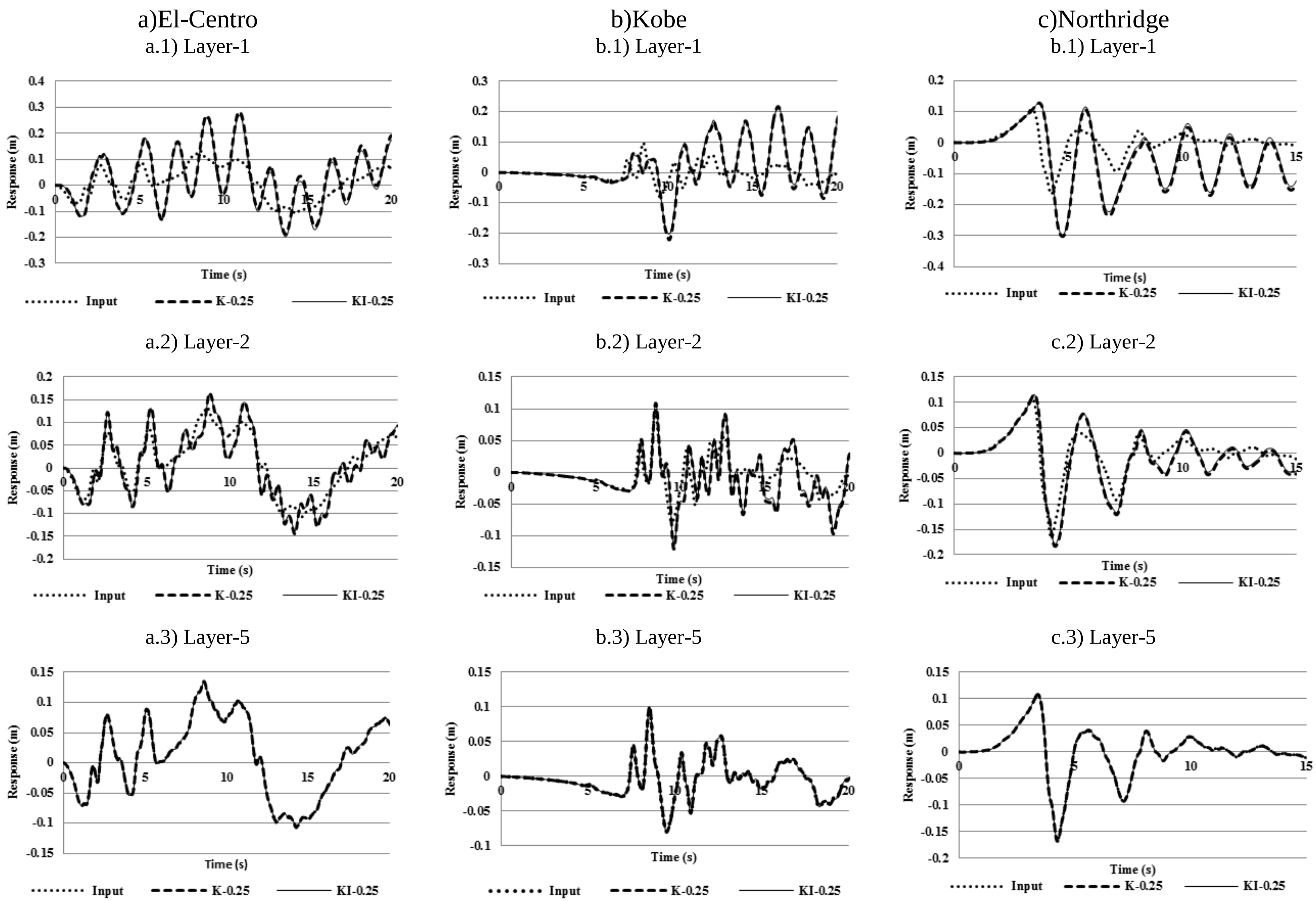


Figure 7: Pile response at mid depth of different layers under a)El-Centro earthquake b) Kobe earthquake c)Northridge earthquake

As the soil stiffness increases with depth, the inertial interaction effects diminish and there is hardly any difference in the response of the pile in the deeper regions under kinematic effects and combined kinematic and inertial effects.

4.2.3 Pile Deflection Patterns

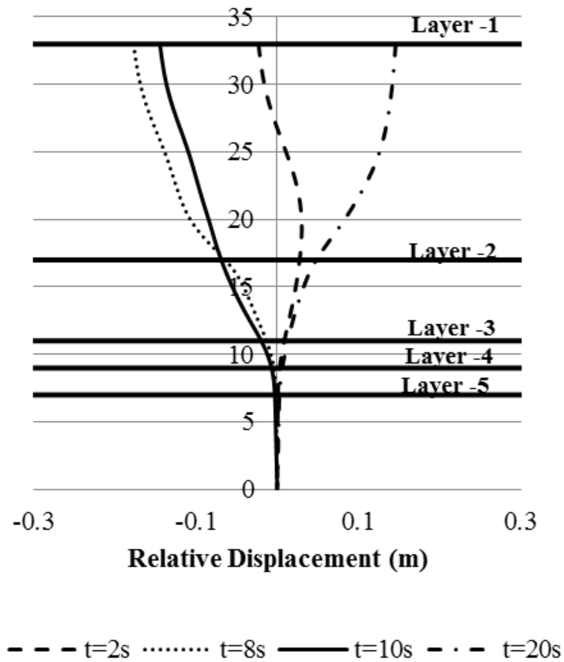
The presence of soil layers with varying stiffness cause different movements in pile along its length in the horizontal direction when subjected to a seismic excitation. The following section describes the different deflection modes that a pile can undergo when subjected to seismic excitations, when they are embedded in layered soil profiles.

When long slender piles are embedded in layered soil, the (horizontal) deflections along the length of the pile can be governed by the stiffness of the surrounding soil layers as described above and results in differences in the deflections along the length of the pile. These differences in pile deflections can excite different deflection modes in the pile which resemble forced vibration modes under the applied seismic excitations. Figure 8 show some of the different deflection modes obtained during the time domain analysis under the three different earthquakes at different times.

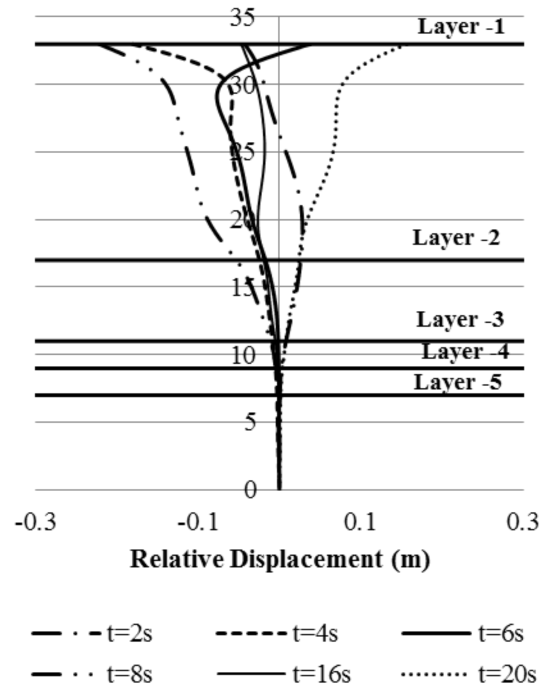
As seen from figure 8, the stiff soil layers do not contribute in generating higher (deflection) modes of the pile and the relative deflections with respect to the pile axis are negligible. The stiff layers, instead provide the fixity for the foundation. When the pile is subjected to the El-Centro earthquake, 1st and 2nd modes are clearly visible, while Kobe and Northridge earthquakes in addition excite the 3rd mode under the kinematic effects only. Even though 2nd and 3rd modes of deflections can be observed during the seismic excitations, the 1st deflection mode is the dominant mode during the excitations when kinematic interaction effects are considered. Incorporation of inertial interaction effects alters the deflection modes and causes the frequent occurrence of higher modes in the pile deflection response. These complex deflection shapes will be a reality in the seismic response of deep piles embedded in layered soils profiles, such as the one considered here.

a)

Kinematic



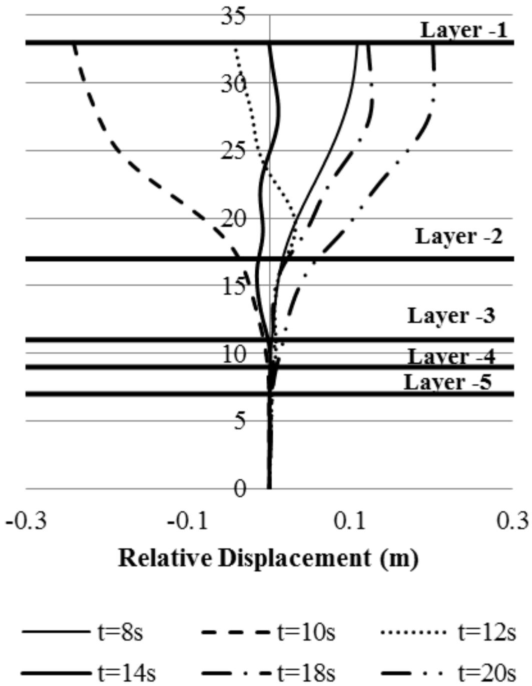
Kinematic + Inertial

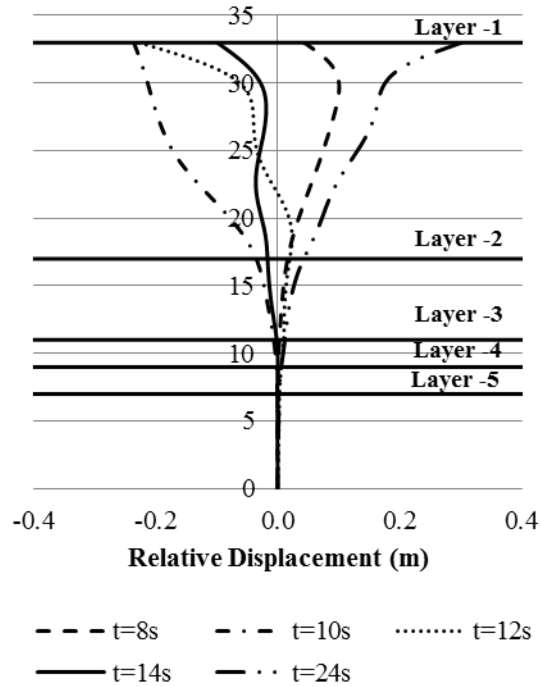


b)

Kinematic

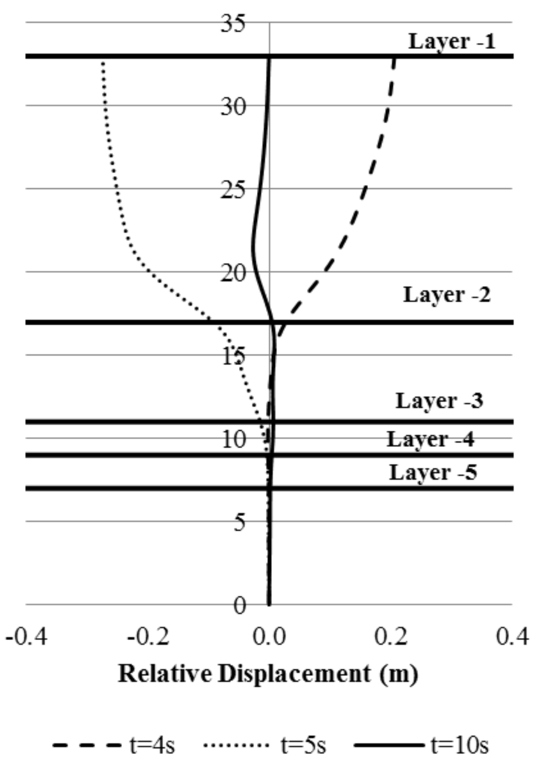
Kinematic + Inertial





c)

Kinematic



Kinematic + Inertial

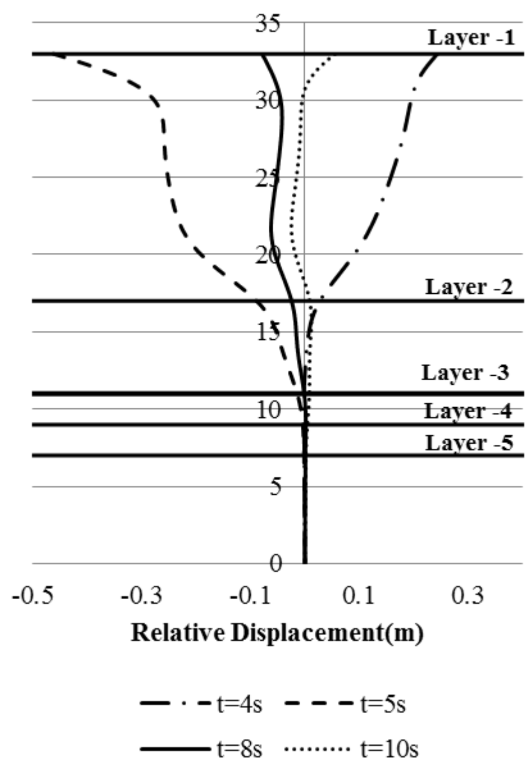


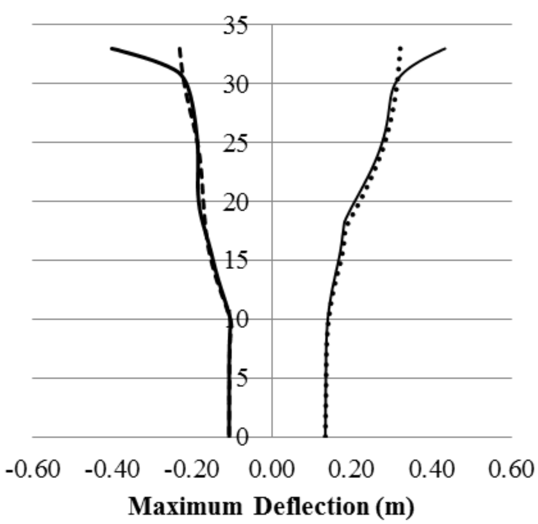
Figure 8: Deflection patterns of the pile when subjected to a)El-Centro earthquake b) Kobe earthquake c)Northridge earthquake

4.2.4 Maximum deflections

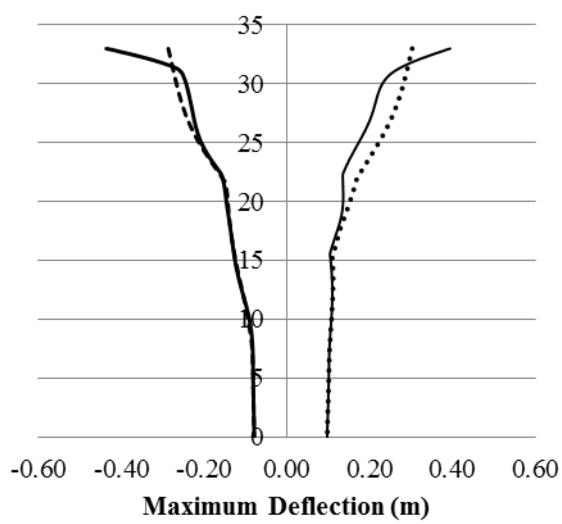
Design of slender piles is usually governed by deflection which should be maintained within the “permissible limits”. Therefore, displacement analysis instead of stress analysis is more appropriate in the design of slender piles in most of the practical situations. In such cases, the maximum pile deflections will be of interest to designers to ensure that the deflections are within “permissible limits”.

Figure 9 shows the deflection envelopes of the pile under (i) kinematic interaction effects and (ii) combined kinematic and inertial effects. According to this figure, maximum deflections due to kinematic effects are almost of similar to the maximum deflections due to the combined kinematic and inertia effects along the pile length except in the upper most 3m length of the pile. In this upper most region of the pile, the maximum deflections are significantly affected by the inertial effects which cause an increase in the maximum deflections. The inertial effects decrease with pile depth (and soil stiffness). In some instances, incorporation of inertial effects can decrease the maximum deflections of the pile in some regions and this behaviour can be observed more clearly in the upper parts of the pile when it is subjected to the Kobe earthquake.

a)



b)



c)

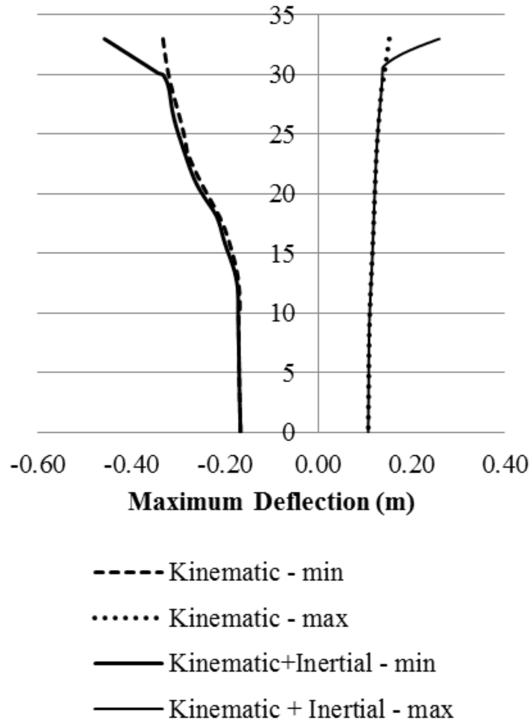


Figure 9: Deflection envelop along the pile length under a)El-Centro earthquake b) Kobe c)Northridge

5.0 Summary and Conclusion

This paper presented a comprehensive three dimensional FE model that can be used to simulate the pile behaviour under a seismic excitation. It has the capability of capturing the combined effects of kinematic and inertial interaction effects. This provides a reliable method of analysis of pile foundation subjected to seismic loads for the engineering practitioners.

The developed method was then extended to investigate the seismic response of a pile foundation in a deep multi-layered soil profile with a soft marine sediment layer at the top. Based on the results following conclusions are drawn.

1. Under kinematic as well as combined kinematic and inertial effects, the seismic response of the pile showed displacements with complex mode shapes. The type of the displacement mode depends on the nature of the soil profile in which the pile is embedded and the inclusion of inertial interaction effects.

2. In all the cases considered in the study, pile head response was mainly governed by the kinematic interaction effects. Addition of inertial effects maintained the response patterns, but increased the head response by about 33% with a small phase lag.

3. The maximum deflections along the pile length are significantly influenced by the inertial interaction effects. Inclusion of inertial interaction increased the maximum deflections in the upper most part of the pile, but its effect diminished rapidly with pile depth. In some

instances however, inclusion of the inertial effects decreased the maximum pile deflections in certain region.

4. The portion of a pile embedded in stiff soil follows the same motion as the input motion applied at the base. However, if a pile is embedded in soft soils, its response deviates significantly from the input motion. This behaviour is also mainly governed by kinematic effects as inertial effects influence only the upper part of the pile foundation.

5. The seismic response of a deep pile embedded in a layered soil profile (such as the one considered herein) can be complex and is influenced by properties of both the soil layers as well as the seismic excitation.

However, these conclusions are limited to the pile, soil and earthquake loading parameters used in this study. This can be extended further to investigate the pile behaviour considering piles nonlinear properties, different soil profiles and different intensities of the seismic loadings.

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7.0 Further Information

*http://edisto.egr.duke.edu/~cc2002web/elcentroN.dat